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A STUDY OF BASE AND BEARING PLATES FOR COLUMNS AND BEAMS

BY

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UNIVERSITY OF ILLINOIS
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ENGINEERING EXPERIMENT STATION

BULLETIN No. 35 JULY 1909

A STUDY OF BASE AND BEARING PLATES FOR COLUMNS AND BEAMS

BY N. CLIFFORD RICKER, PROFESSOR OF ARCHITECTURE

CONTENTS

I. Introduction

The primary object of this study has been to produce a series of accurate formulas and tables for the different forms and materials of base and bearing plates. These formulas are required to be as simple and as easily applied as possible and to be in accordance with the local building ordinances of the larger cities in the United States.

A secondary purpose has been to devise a similar series of formulas based on the common theory of the fracture of such plates and to check the accuracy of these common formulas by experimental tests of a series of plates designed in accordance with such formulas. A number of typical plates were so designed and tested in 1907 by Mr. C. R. Dick, B. S. in Architectural Engineering, and the results were discussed in his thesis.

Up to the present time, very little study of theory and no experimental research appear to have been devoted to these plates. Even the German writers usually give incorrect theories with formulas based thereon, formulas which give erroneous dimensions of base plates.

II. Limit of Safe Pressure of Plate on Masonry

The maximum safe pressure of the plate on the masonry beneath it varies greatly, according to the nature and the resistance of this material; the requirements of the city building ordinances also differ considerably for the same kind of masonry. These requirements seem to be based upon local customs and not on actual experimental tests. Examples of such maximum safe pressures are here quoted from the ordinances of New York, Chicago and Washington, D. C., selected as representative cities, and these are further compared with the values given in Kidder's Pocket Book. (Table 1.)

Therefore this maximum safe pressure of the plate on masonry appears to vary between 70 and 1000 pounds per square inch, the larger value being allowed for truly dressed large blocks of stone. The value to be employed must be taken in accordance with the local building ordinance.

TABLE 1

Safe Pressures on Masonry in Pounds per Square Inch

Masonry	*N. Y.*	*Chicago*	*Wash'ton*	*Kidder*
Granite	......	173.61	1000–2400	1000
Limestone	......	173.61	700–2300	....
Sandstone	......	173.61	400–1600	400–700
Dimension stone, rough	......	138.89	....	....
Rubble in portland cement	138.89	......	140.	150–200
Rubble in natural cement	111.11	......	111.	....
Rubble in cement and lime	97.22	......	97.	....
Rubble in lime	94.44	......	70.	....
Brickwork in portland cement	208.33	173.61	250.	150–250
Brickwork in natural cement	....	125.	208.	....
Brickwork in cement and lime	159.72	......	160.	....
Brickwork in lime	111.11	90.28	111.	100–120
Concrete in portland cement	208.33	173.61	208–230	200.
Concrete in natural cement	111.11	......	111–125	....

III. Maximum Safe Fiber Stress in Metal of Plate

This maximum fiber stress in pounds per square inch occurs at either top or bottom of a plate of uniform thickness, or at the bottom of a cast-iron plate of tapered thickness. It must not exceed the intensities given in the following table, which are almost uniformly adopted throughout the United States.

TABLE 2

Maximum Safe Fiber Stress in Plates

Metal	*N. Y.*	*Chicago*	*Wash'ton*	*Kidder*
Steel, compression	16 000	16 000	16 000	16 000
Steel, tension	16 000	16 000	16 000	16 000
Wrought Iron, compression	12 000	12 000	12 000	12 000
Wrought Iron, tension	12 000	12 000	12 000	12 000
Cast Iron, compression	16 000	10 000	16 000	......
Cast Iron, tension	3 000	2 500	3 000	......

Evidently the maximum tensile stress in cast-iron permitted in Chicago might safely be increased from 2500 to 3000 pounds per square inch, which is permitted in about one-half the cities in the United States.

IV. Common Formulas

The ordinary formulas for base plates are usually empirical (Kidder) or they are otherwise based on the theory that the line of fracture of a base plate is a straight line tangent to the exterior of the foot of the column standing on the plate (Kohnke). In accordance with this theory, a series of formulas was deduced and later published in the Handbook of the Chicago Architects' Business Association. The essential formulas are the following:

A. For Steel Plates of Uniform Thickness

Let p = maximum safe pressure of plate on masonry in lb. per sq. in.
k = perpendicular distance in inches from column to edge of plate.
t = thickness of plate required, in inches.

(a) For square plates (Fig. 1)

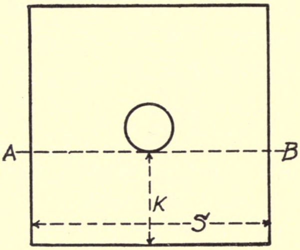


Fig. 1

The line AB (Fig. 1) is the theoretical fracture line.

Then $$t = \frac{k}{40}\sqrt{\frac{p}{10}} \qquad (1)$$

(b) For octagonal plates (Fig. 2)

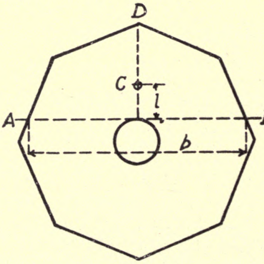


Fig. 2

The segment ADB may be divided into triangles, when its center of gravity C is easily found by graphical methods. (See Graphic Statics.)

Let l = perpendicular distance in inches from C to line of fracture AB.

b = length in inches of line of fracture.

a = area in square inches of the segment ADB outside the line AB.

Then
$$t = \frac{1}{40}\sqrt{\frac{3\,apl}{5\,b}} \tag{2}$$

(c) For circular plates (Fig. 3)

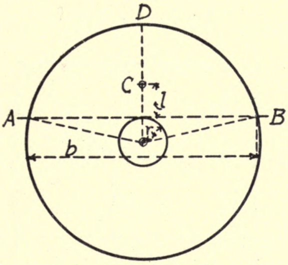


FIG. 3

Join the ends A and B of the chord fracture line with the center O by the radii AO, OB, and draw OD perpendicular to AB.

Let β = angle AOB in degrees between radii AO, BO.

A = area in square inches of the entire circle of the plate.

b = length in inches of chord fracture line AB.

R = external radius of end of column in inches.

Then $\frac{A\,\beta}{360}$ = area of the sector $ADBO$ in square inches.

$\frac{bR}{2}$ = area in square inches of triangle ABO.

Hence $\frac{A\,\beta}{360} - \frac{bR}{2} = a$ = area in square inches of segment ADB.

Also $\frac{b^3}{12\,a}$ = distance in inches from center O to center of gravity C of the segment.

Hence $l = \frac{b^3}{12\,a} - R$ = perpendicular distance in inches from C to chord line of fracture AB.

Finally,
$$t = \frac{1}{40}\sqrt{\frac{3\,apl}{5\,b}} \tag{3}$$

B. For Cast-iron Plates of Tapered Thickness

Such plates are of uniform thickness only beneath the end of a column or beam, and are flat on the under side, but are beveled off on top from column to edge of the plate. These edges are usually made at least ⅜-in. thick, and a good rule is to make the edge one-fourth the thickness at the middle. These formulas are deduced for sharp edges or a trapezoidal fracture section, and they are therefore only approximate for plates with thick edges.

(*a*) For square plates (Fig. 4)

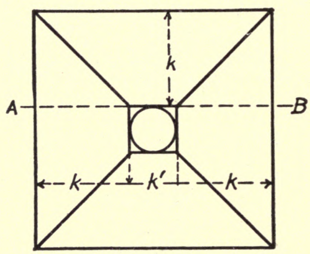


FIG. 4

Let k = projection of the edge of the plate outside the column, measured in inches and perpendicular to the edge.

k' = side in inches of square on top of plate and tangent to column.

The line of fracture AB is parallel to a side.

The thickness in inches of plate at middle is then given by the formula

$$t = \frac{k}{50}\sqrt{\frac{6\,p\,(k + \frac{k'}{2})}{k + k'}} \qquad (4)$$

(*b*) For octagonal plates (Fig. 5)

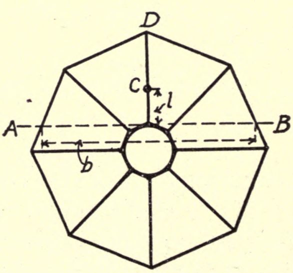


FIG. 5

In the manner already explained for the octagonal steel plate may be found the area a of the segment ADB, its center of gravity C, and the distance l.

The thickness in inches at the middle is then approximately

$$t = \frac{1}{50}\sqrt{\frac{12\,apl}{b}} \qquad (5)$$

(c) For circular plates (Fig. 6)

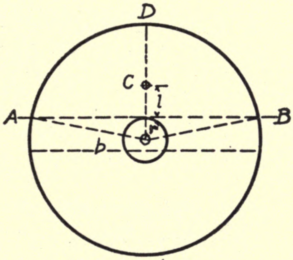


FIG. 6

As for circular steel plates:

$a = \dfrac{A\,\beta}{360} - \dfrac{bR}{2} =$ area in square inches of segment ADB.

$l = \dfrac{b^3}{12\,a} - R =$ perpendicular distance in inches from C to line AB. The actual fracture section lying between a very flat hyperbola and its chord, a parabola may be substituted therefor without material error.

The thickness at middle is given by the formula

$$t = \frac{1}{50}\sqrt{\frac{35\,apl}{8\,b}} = \frac{1}{23.9}\sqrt{\frac{apl}{b}} \qquad (6)$$

V. Results of Tests by C. R. Dick

Employing the preceding formulas, Mr. C. R. Dick designed in 1907 a series of square, octagonal and circular plates of steel and of cast iron, and afterwards tested them in the testing laboratory of the University.

Each plate had a bottom area of 400 sq. in. and transmitted the very moderate safe pressure of 50 lb. per sq. in., making a total maximum safe pressure of 20 000 lb. for the entire plate. The thickness of each plate was made such that the maximum safe fiber stress in the fracture section produced by this maximum safe pressure should not exceed 16 000 lb. for steel or 2500 lb. for cast iron in tension, as required by the Chicago ordinance.

The distribution of the pressure of the plate uniformly over the lower surface required some form of elastic cushion between the plate itself and the very rigid bed of the testing machine. A cushion was composed of several folded blankets, a folded woollen comfortable and two thicknesses of rubber packing, but it failed under moderate pressures, though not sufficiently to seriously injure the plates except in

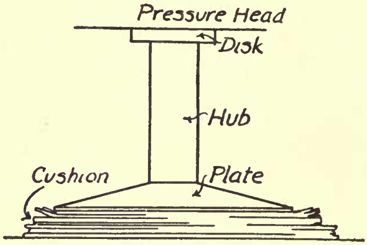


Fig. 7

the case of the cast iron. (Fig. 7). A cushion of dry sand forming a layer 2 inches thick was enclosed within a steel hoop a little larger than the plate, but the sand packed irregularly and failed to transmit a uniform pressure. A satisfactory cushion was finally composed of 11 layers of oak pieces, cut 24 x 3 x ⅞ in. piled in crosswise layers, leaving ¼ in. spaces between the pieces to permit expansion. (Fig. 8). This cushion proved to be sufficiently elastic and also able to sustain pressures sufficient to break the cast-iron plates. Indeed it later supported without great injury a load of 620 000 lb., or 31 times the safe pressure for which the plates were designed. Any injured pieces could easily be replaced in order to maintain the efficiency of the cushion.

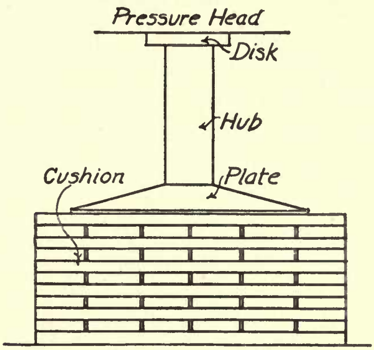


FIG. 8

Pressure was applied to the flat top of the plate by a hollow cylindrical cast-iron hub 12 inches long, 4 inches external diameter, and ¾ inch thickness of metal. Since the plates were not planed on top (which is seldom done in practical construction), several thicknesses of heavy manila paper were inserted between the hub and the plate. On the hub rested the pressure head of the testing machine, while the elastic cushion was placed between the plate and the bed of the machine. Thus the conditions of the test fairly represented those existing in actual structures, where the masonry yields somewhat and is not absolutely rigid like the bed of the testing machine.

The square steel plates were $^{13}/_{16}$ in. thick; the octagonal and circular steel plates were ¾ in. thick. The upward deflections of their outer edges were measured at 4 points to $^{1}/_{1000}$ in., and each plate was gradually loaded to 120 000 lb., or with six times its maximum safe load. This produced an average maximum deflection of 0.29 in., leaving a permanent set averaging 0.18 in., when the pressure was removed. The plate was thus dished, but no failure or cracks were produced.

These results show that a steel base plate bends more than one of cast iron, and that it does not so uniformly distribute its load over the masonry beneath it as a more inflexible cast-iron plate. It follows that the latter plate is preferable for the purpose. Empirical rules usually make steel plates one-half the thickness of cast-iron plates supporting equal loads.

Mr. Dick deduced from his experiments the following conclusions for steel plates.

"1. The (preceding) formulas for the design of steel base plates are entirely safe.

2. The limit of 16 000 pounds fiber stress permitted by the Chicago ordinance is perhaps too large, since marked deflections take place rapidly after this fiber stress has been exceeded.

3. Steel plates projecting more than two diameters of the hub (or column) beyond it, should be designed for deflection, or it would be better to use a cast-iron plate for large loads.

4. The circular is the most economical shape for a bearing plate."

The cast-iron plates were beveled on top from the hub to a uniform thickness of $\frac{3}{8}$ in. at the edges, the bottom being a plane surface. They were cast and tested in sets of threes of each form. The square plates (Fig. 9) were $2\frac{9}{16}$ inches thick beneath the hub; the octagonal plates (Fig. 10) were $2\frac{1}{8}$ inches, while the circular plates (Fig. 11) were $2\frac{3}{16}$ inches. All cast-iron plates were tested to fracture, which occurred with the load indicated in the figures for each plate. Plates *A* 1, *B* 1 and *C* 1 were tested on the cushion of folded blankets, etc. (Fig. 7), which crushed and distributed the pressure unequally, the plate thus failing under a smaller load. The other plates were tested on the wooden cushion, Fig. 8.

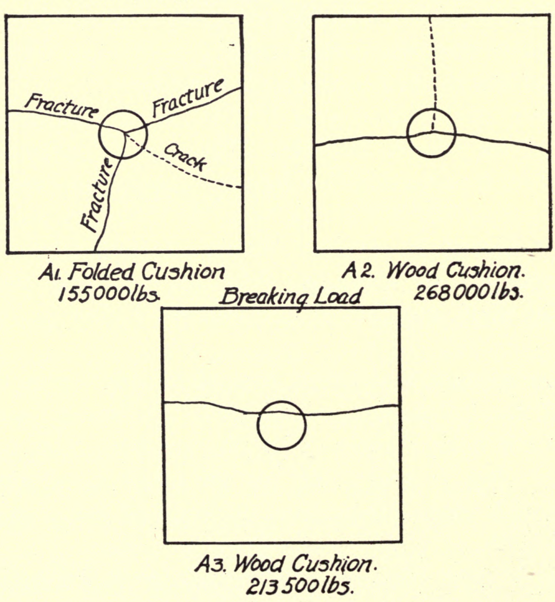


FIG. 9

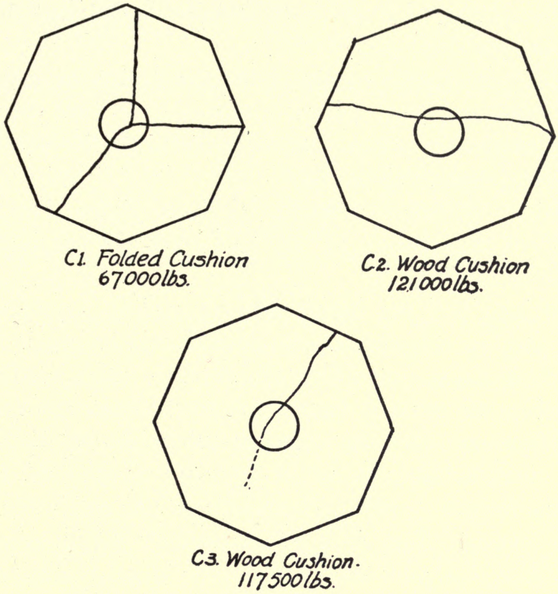

C1. Folded Cushion
67000 lbs.
C2. Wood Cushion
121000 lbs.
C3. Wood Cushion.
117500 lbs.

Fig. 10

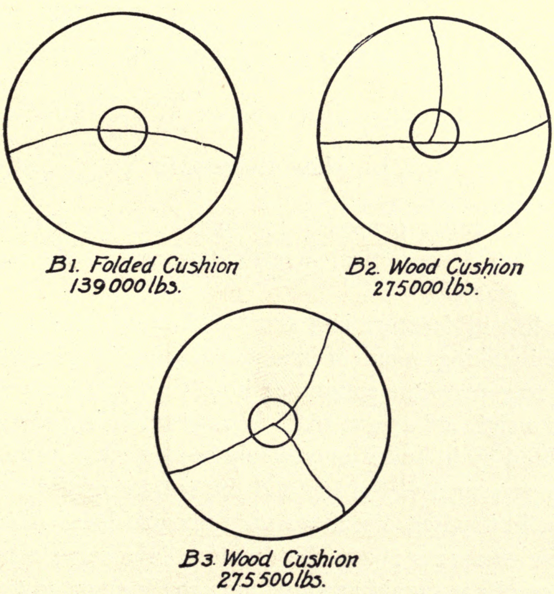

B1. Folded Cushion
139000 lbs.
B2. Wood Cushion
275000 lbs.
B3. Wood Cushion
275500 lbs.

Fig. 11

From the results of these tests, Mr. Dick deduced the following conclusions for cast-iron plates.

"1. The (preceding) formulas for the design of cast-iron plates may be used with safety.

2. A greater fiber stress than that permitted by the Chicago ordinance could be used with safety.

3. Cast iron is better adapted for base plates than steel, as it gives a uniform distribution of the load over the bearing area for a greater range of loading.

4. Cast iron will not deteriorate as rapidly as steel when in a damp place, and for this reason cast iron should be preferred."

Mr. Dick likewise notes that the fracture lines pass through the center of each plate and are not tangent to the hub, as assumed by the preceding formulas. He also suggests that assuming the fracture line through the center of each plate, the resultant moment of the pressures about this line may be found and equated to the resistance moment of the fracture section. But since this procedure would render the formulas much more complex and increase the labor of designing a plate, the extra labor would not be repaid, being unnecessary for safety.

The direction of the fracture line in the plates tested was sometimes changed by the influence of side cracks, probably due to slight irregularities in the distribution of the pressure, or perhaps to slight flaws in the castings.

VI. New Formulas

The new formulas proposed are based on the following principles, the result of a theoretical investigation and of the nature of the failures of the plates.

1. The line of fracture is a shorter diameter of the plate.

2. The breaking moment about this line is greater than that about a line tangent to the column or hub.

3. The weight of a base plate is very small in proportion to the load transmitted by it, and it may safely be neglected in the formulas.

4. The pressures at the top and the bottom of the plate are then equal and act in opposed directions with unequal lever arms.

5. Their moments about the fracture line being necessarily unequal, their resultant maximum safe moment is equal to the maximum safe resisting moment of the fracture section of the plate.

VII. Resultant Bending Moment Acting About Fracture Line

Let $A =$ total area of the plate in square inches;
$P =$ total load in pounds transmitted by it;
$L' =$ perpendicular distance in inches from fracture line to center of upward pressures on one-half the area of the plate;
then $L' = 0.25 \times$ length in inches of a wall or bearing plate;
$= 0.25 \times$ side of a square plate;
$= 0.2187 \times$ inscribed diameter of octagonal plate;
$= 0.2122 \times$ diameter of circular plate.
Let $L'' =$ perpendicular distance in inches from fracture line to center of downward pressures on one-half the area of the plate;
then $L'' = 0.25 \times$ width of a beam or girder on a bearing plate;
$= 0.25 \times$ side of a solid square post or column;
$= 0.2187 \times$ diameter of solid octagonal post or column;
$= 0.2122 \times$ diameter of solid cylindrical post or column.
Let $R =$ radius of outer circle inscribed in cross section of hollow square or cylindrical column;
$r =$ radius of inner circle inscribed in hollow square or round column;

Then $L'' = 0.500 \dfrac{R^3 - r^3}{R^2 - r^2}$, for a hollow square column.

Limiting values are $\begin{cases} 0.500R \text{ for a solid column.} \\ 0.750R \text{ for a very thin shell column.} \end{cases}$

Also $L'' = .424 \dfrac{R^3 - r^3}{R^2 - r^2}$, for a hollow cylindrical column.

Limiting values are $\begin{cases} .424R \text{ for a solid column.} \\ .637R \text{ for a very thin shell column.} \end{cases}$

These values of L'' may readily be found by calculation, but much more easily by the aid of the graphical table, Fig. 12.

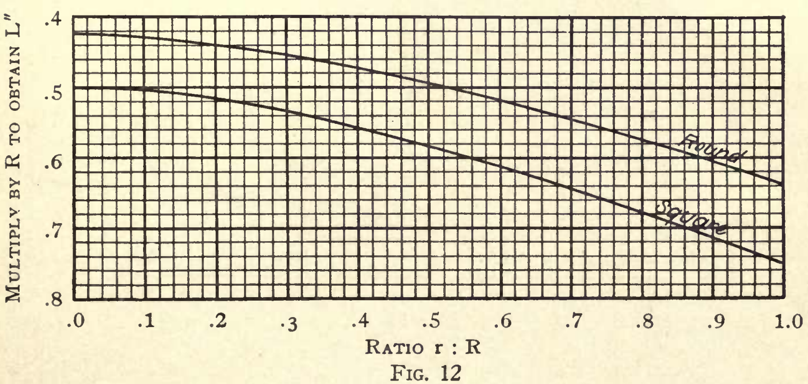


Fig. 12

Example. Assume a hollow cylindrical column, 12 in. external and 9 in. internal diameter.

Then $$\frac{r}{R} = \frac{4.5}{6.00} = 0.75.$$

By the table, Fig. 12, $L'' = 0.56\,R = 3.36$ in.

VIII. Resistance Moment of Fracture Section

The fracture section is a plane vertical section through the fracture line.

Let f = maximum safe fiber stress in pounds per square inch.

B = length in inches of fracture section.

t = thickness in inches of plate at center under column.

I = moment of inertia of the fracture section.

c = vertical distance in inches from horizontal neutral axis to bottom of fracture section.

Then $\dfrac{fI}{c}$ = maximum safe resistance moment of fracture section in general.

(a) For a rectangular fracture section of plate of uniform thickness,

$$\frac{fBt}{6} = \text{maximum safe resistance moment.}$$

(b) For tapered section with sharp edges. (Fig. 13.)

Let k = length in inches of horizontal top of section under column or diameter of column.

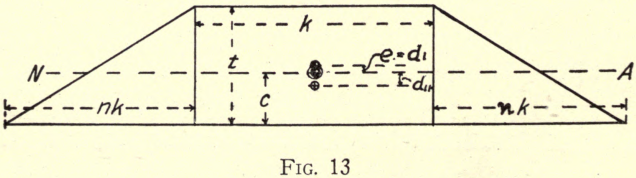


Fig. 13

nk = length in inches of each tapered portion of the fracture section.

$B = k\,(2n + 1)$ = length in inches of fracture section.

$A' = kt$ = area in square inches of rectangle in Fig. 13.

$A'' = nkt$ = area in square inches of both triangles in Fig. 13.

$d' = \dfrac{nt}{6\,(n+1)}$ = vertical distance in inches from center of gravity of rectangle to neutral axis of fracture section, Fig. 13.

$d'' = \dfrac{t}{6} - \dfrac{nt}{6\,(n+1)}$ = vertical distance from center of combined triangles to neutral axis of fracture section, Fig. 13.

$c = d'' + \dfrac{t}{3}$ = vertical distance in inches from neutral axis to bottom of fracture section.

Also $I = \dfrac{kt^3}{12} + ktd'^2$ = moment of inertia for rectangle.

$I = \dfrac{nkt^3}{18} + nktd''^2$ = moment of inertia for combined triangles.

Finally $\dfrac{fI}{c} = \dfrac{fk}{c}\left[\dfrac{t^3}{12} + td'^2 + \dfrac{nt^3}{18} + ntd''^2\right]$ = safe moment of resistance of fracture section of the plate. (7)

(c) For tapered section with thick edges (Fig. 14)

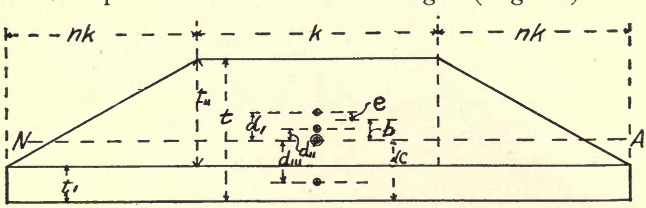


FIG. 14

Divide the fracture section into two rectangles and two triangles as in the figure.

Let t = thickness in inches at the middle of the plate.

t' = thickness in inches at the edges.

$t'' = t - t'$ = thickness of the taper in plate.

Then $A' = kt''$ = area in square inches of the upper rectangle.

$A'' = nkt''$ = area in square inches of combined triangles.

$A''' = (2n+1)\,kt' =$ area in square inches of lower rectangle.

$e = \dfrac{nt''}{6(n+1)} =$ vertical distance in inches from center of gravity of upper rectangle to joint center of gravity of this rectangle and combined triangles.

$$b = \left(\frac{t}{2} - e\right)\frac{A'''}{A' + A'' + A'''}$$

$= \left(\dfrac{t}{2} - e\right)\left[\dfrac{(2n+1)\,t'}{t'' + nt'' + (2n+1)t'}\right] =$ vertical distance in inches from the preceding joint center of gravity to the neutral axis of fracture section.

Then $d' = e + b =$ vertical distance in inches from center of gravity of upper rectangle to neutral axis of fracture section.

$d'' = d' + \dfrac{t''}{6} =$ vertical distance from center of gravity of combined upper rectangle and both triangles to neutral axis of fracture section.

$d''' = \dfrac{t}{2} - d' =$ vertical distance in inches from center of gravity of lower rectangle to neutral axis of section.

Then $I' = \dfrac{kt''^3}{12} + kt''d'^2 =$ moment of inertia of upper rectangle about the neutral axis.

$I'' = \dfrac{nkt''^3}{18} + nkt''d''^2 =$ moment of inertia of combined triangles about the neutral axis.

$I''' = \dfrac{(2n+1)}{12}\,kt'^3 + (2n+1)\,kt'd'''^2 =$ moment of inertia of lower rectangle about the neutral axis.

Finally, summing these results for moment of inertia of entire section:

$$\frac{fI}{c} = \frac{fk}{c}\left[\frac{t''^3}{12} + td'^2 + \frac{nt''^3}{18} + nt''d''^2 + \frac{(2n+1)\,t'^3}{12} + (2n+1)\,t'd'''^2\right] =$$

resistance moment of fracture section of a plate with thick edges $= t'$. (8)

IX. Method of Approximation

As cast-iron plates are generally made with thick edges, the practical application of the last formula is quite tedious. In order to apply it, it becomes necessary first to assume the thicknesses t, t' and t'', then using the preceding formulas to determine the corresponding safe resistance moment of the fracture section. This process must doubtless be repeated several times before a plate is found which has the required safe moment of resistance of section. Hence the necessity for a simplification of the method by directly obtaining the required values of the thicknesses t, t' and t'', which is accomplished in the following manner.

By the formula for a rectangular fracture section, the values of

$$\frac{I}{c} = \frac{bt^2}{6} = \frac{kt^2}{6}(2n + 1) \tag{9}$$

are computed for $n = 0$, $n = 1$, $n = 2$, $n = 3$, $n = 4$, $n = 5$, and these values lie in a straight line, when they are plotted as in the uppermost line in Fig. 15.

By formula (7) for tapered plates with sharp edges, are next computed the values of $\frac{I}{c}$ corresponding to $n = 0$, $n = 1$, $n = 2$, $n = 3$, $n = 4$, and $n = 5$. These values are then plotted in Fig. 15, forming the slightly curved and dotted line, next to the lower straight line. Joining the ends of the curve by the straight full line, this is found to almost coincide with the curved line, for which it may be substituted with a slight error on the safe side.

Similarly for $n = 5$ only, $\frac{I}{c}$ is computed for edge thickness $t' =$ $0.1\,t$, $0.2\,t$, $0.3\,t$, $0.4\,t$, $0.5\,t$, $0.6\,t$, $0.7\,t$, $0.8\,t$, and $0.9\,t$. These values are laid off on the vertical for $n = 5$, and straight radial lines are drawn to these points in Fig. 15. The corresponding per cents of resistance may then be easily computed for $n = 5$, as written in Fig. 15, and which signify that the resistance moment of a tapered fracture section is a certain per cent of that of a rectangular section of equal thickness, only for $n = 5$.

It now remains to determine this per cent for any other value of n.

The graphical table in Fig. 16 is readily computed and plotted from the data obtained in Fig. 15. It is used as follows:

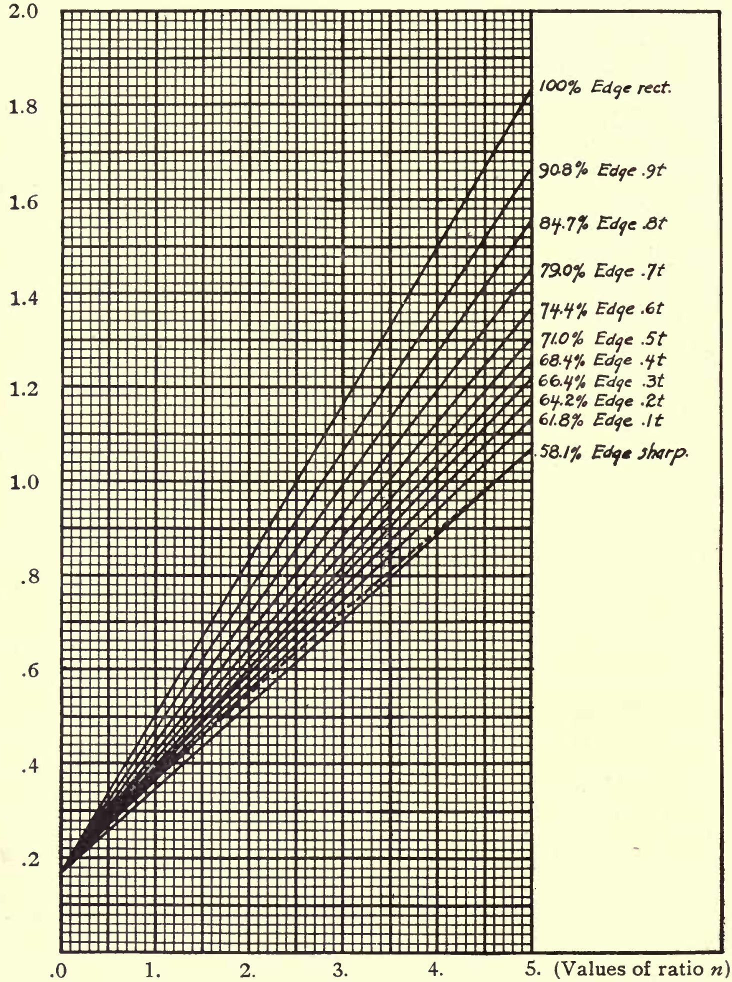


Fig. 15

Let @ = per cent of resistance moment of a rectangular fracture section, which is possessed by a tapered section with the same thickness t and the same value of n.

Example 1. Let $t' = 0.3\ t$ = edge thickness and $n = 2$.

A vertical through 2 in Fig. 16 intersects the curve 0.3 on a horizontal through 70.5 at the left, which is the required per cent.

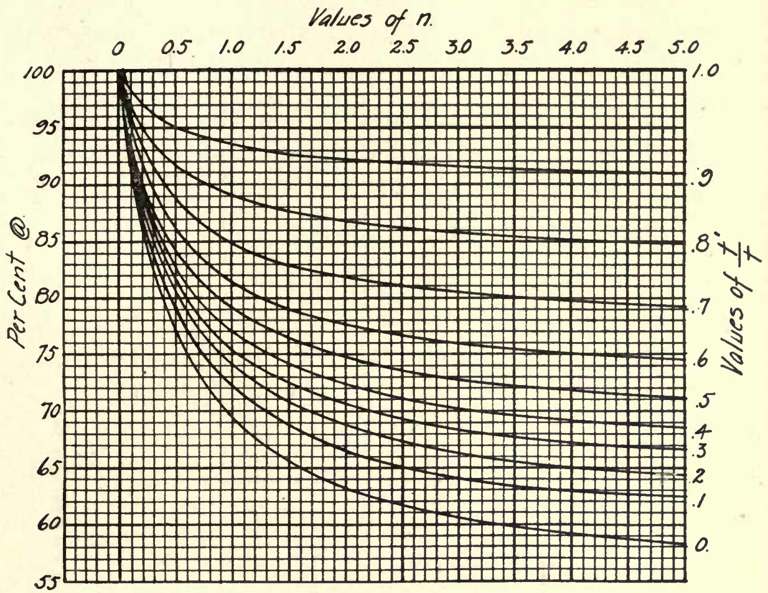


FIG. 16

Example 2. Let $t' = 2.5$ and $n = 3.5$.

A vertical through 3.5 intersects the interpolated curve 2.5 on a horizontal through 70.2 at the left, the required per cent.

It is further evident that the actual thickness t of a tapered plate must be somewhat greater than that of a plate of uniform thickness, when both are required to possess equal resistance moments.

The general formula for base plates is:

$$t = \sqrt{\frac{3\,P}{fB}(\,L' - L'')} =$$

thickness in inches for a plate of uniform thickness. (10)

Or $$t = \frac{1}{50}\sqrt{\frac{3\,P}{B}\,(\,L' - L''\,)} =$$

thickness for a cast-iron plate. (11)

Let @ = per cent corresponding to $\frac{t'}{t}$ and n, found by table (Fig. 16)

Then for cast iron, $t = \sqrt{\frac{100}{@}}\,\frac{1}{50}\sqrt{\frac{3\,P}{B}\,(\,L' - L''\,)} =$ required thickness t under the column for a tapered cast-iron plate, possessing a resistance moment equal to that of a plate of uniform thickness t determined by formula 11. (12)

X. Complete Formulas for Plates of Uniform Thickness

By equating the resultant bending and safe resistant moments acting about the neutral axis of the fracture section of the plate, the following general formulas are obtained:

The primary formula is:

$$M = \frac{P}{2}(L' - L'') = R = \frac{fI}{c}. \tag{13}$$

(a) Bearing plate on wall (Fig. 17)

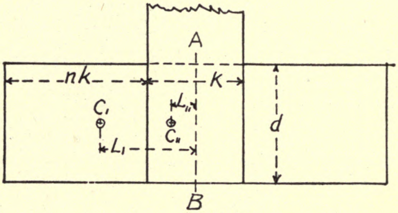


Fig. 17

Let d = width in inches of the plate, usually equal to the thickness of the wall.

Then

$$\frac{P}{2}(L' - L'') = \frac{fdt^2}{6}.$$

Or

$$\frac{Pnk}{4} = \frac{fdt^2}{6}.$$

Hence $t = \sqrt{\dfrac{3\,Pnk}{2\,fd}}$ = thickness of the plate in inches. (14)

And $t = \dfrac{1}{103.3}\sqrt{\dfrac{Pnk}{d}}$ = thickness in inches of a steel plate. (15)

$t = \dfrac{1}{44.7}\sqrt{\dfrac{Pnk}{d}}$ = thickness of cast-iron plate, $f = 3000$ lb. (16)

$t = \dfrac{1}{40.8}\sqrt{\dfrac{Pnk}{d}}$ = thickness of cast-iron plate, $f = 2500$ lb. (17)

If a square or cylindrical column stands on the bearing plate instead of the end of a beam resting thereon, L'' is to be found and then inserted in the general formula, by which t may then be found. (Sec. VII). The general formulas then become:

$$t = \frac{1}{103.3}\sqrt{\frac{P}{d}\left(\frac{nk}{2} + \frac{k}{4} - L''\right)} = \text{thickness for steel plate.} \qquad (18)$$

$$t = \frac{1}{44.7}\sqrt{\frac{P}{d}\left(\frac{nk}{2} + \frac{k}{4} - L''\right)} = \text{thickness for cast-iron, } f = 3000 \text{ lb.} \qquad (19)$$

$$t = \frac{1}{40.8}\sqrt{\frac{P}{d}\left(\frac{nk}{2} + \frac{k}{4} - L''\right)} = \text{thickness for cast-iron, } f = 2500 \text{ lb.} \qquad (20)$$

Formulas 16, 17, 19 and 20 are also applicable to cast-iron bearing plates tapered in thickness from beam or column to each end, since the fracture section always remains rectangular in form.

(b) Square plate (Fig. 18)

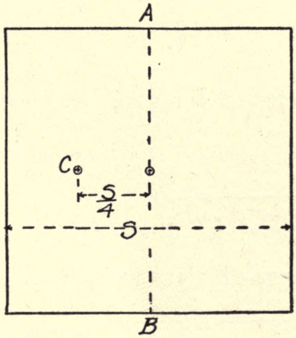


FIG. 18

Let $S =$ side of square plate in inches.

$$\text{Then } t = \sqrt{\frac{3P}{fS}\left(\frac{S}{4} - L''\right)} = \text{thickness of plate in inches.} \qquad (21)$$

$$t = \frac{1}{73}\sqrt{\frac{P}{S}\left(\frac{S}{4} - L''\right)} =$$

thickness of steel plate in inches. (22)

$$t = \frac{1}{31.6}\sqrt{\frac{P}{S}\left(\frac{S}{4} - L''\right)} =$$

thickness for cast-iron, $f = 3000$ lb. (23)

$$t = \frac{1}{28.9}\sqrt{\frac{P}{S}\left(\frac{S}{4} - L''\right)} =$$

thickness for cast-iron, $f = 2500$ lb. (24)

(*c*) Octagonal plate (Fig. 19)

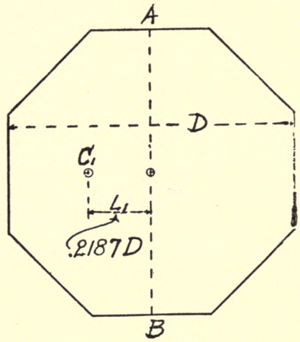


FIG. 19

Let D = inscribed diameter of octagon.

$L' = .2187D.$

$$t = \sqrt{\frac{3\,P}{fD}\left(.2187D - L''\right)} =$$

thickness of plate in inches. (25)

$$t = \frac{1}{73}\sqrt{\frac{P}{D}\left(.2187D - L''\right)} =$$

thickness of steel plate. (26)

$$t = \frac{1}{31.6}\sqrt{\frac{P}{D}\left(.2187D - L''\right)} =$$

thickness of cast-iron plate, $f = 3000$ lb. (27)

$$t = \frac{1}{28.9}\sqrt{\frac{P}{D}\left(.2187D - L''\right)} =$$

thickness of cast-iron plate, $f = 2500$ lb. (28)

(*d*) Circular plate (Fig. 20)

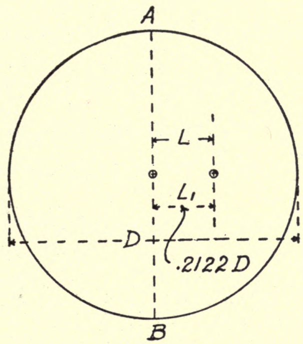


FIG. 20

Let D = diameter of the plate in inches.

$L' = .2122D$.

$$t = \sqrt{\frac{3\,P}{fD}\left(.2122D - L''\right)} =$$

thickness of the plate. (29)

$$t = \frac{1}{73}\sqrt{\frac{P}{D}\left(.2122D - L''\right)} =$$

thickness for steel plate. (30)

$$t = \frac{1}{31.6}\sqrt{\frac{P}{D}\left(.2122D - L''\right)} =$$

thickness for cast-iron, $f = 3000$ lb. (31)

$$= \frac{1}{28.9}\sqrt{\frac{P}{D}\left(.2122D - L''\right)} =$$

thickness for cast-iron, $f = 2500$ lb. (32)

XI. For Plates of Tapered Thickness

These comprise all cast-iron base plates, which are always made thinner at their edges for the sake of economy. Bearing plates of cast iron set on walls, however, have a rectangular fracture section, for which direct formulas have already been given. (Formulas 16, 17, 19, 20.)

The thickness required at the middle of a tapered cast-iron plate may now be easily found.

1. Compute the thickness for a rectangular fracture section.
2. Assume the desired ratio $\frac{t'}{t}$ of thickness at edge and middle.
3. By the table, Fig. 16, determine the per cent @ of its resistance in comparison with a plate of rectangular section of equal width and thickness.
4. By formula 12 compute the required actual thickness t at the middle of the plate. Then t' is easily found.

Example. A hollow cylindrical cast-iron column has an external diameter of 6 in., is ¾-in. thick and transmits a load of 60 000 lb. to the plate. Required the safe dimensions of a square cast-iron base-plate; fiber stress 2500 lb. per sq. in.; ratio $\frac{t'}{t} = 0.15$.; safe resistance of masonry under plate = 125 lb. per sq. in.

$$\sqrt{\frac{60\,000}{125}} = 21.91 \text{ in.} = \text{side of plate required.}$$

$$nk = \frac{21.91 - 6.}{2} = 7.96 \; ; \quad n = \frac{7.96}{6.00} = 1.33.$$

For this column, $R = 3.00$, $r = 2.25$; $\frac{r}{R} = 0.75$.

By the table, Fig. 12, $L'' = 0.56$, $R = 1.68$ in.

By formula 24,

$$t = \frac{1}{28.9}\sqrt{\frac{60\,000}{21.91}\left(\frac{21.91}{4} - 1.68\right)} = 3.53 \text{ in.}$$

By the table, Fig. 16, for $\frac{t'}{t} = 0.15$ and $n = 1.33$, @ = 71 per cent.

By formula 12,

$$t = \sqrt{\frac{100}{71}} \times 3.53 = 4.19 \text{ in.} = \text{the required middle thickness,}$$

And $4.19 \times 0.15 = 0.63$ in. = thickness at the edge of the plate.

XII. Graphical Tables

It is evident from the example just worked out, that the calculations required for any particular base plate are quite simple and rapid, particularly if 4-place logarithms are employed. It is, however, entirely possible to devise a series of graphical tables for materially reducing this labor, thus saving valuable time and lessening liability to errors.

XIII. Table for Dimensions of Base Plates

Fig. 21 is a graphical table for determining by inspection the side of a square, or the diameter of an octagonal or round plate, required to safely transmit loads not exceeding 200 tons (400 000 lb.), allowing safe pressures of the plate on masonry between 50 and 250 lb. per sq. in. Fig. 22 is merely an enlarged portion of the same table for loads not exceeding 20 tons.

By the use of the upper scale corresponding to the shape of the plate, it is possible to employ this single table for square, octagonal and circular plates, by reading the required side or diameter on the proper scale.

Example. A plate is required to safely transmit a load of 100 tons, allowing a safe pressure of 125 lb. per sq. in. on the masonry beneath it.

Locating the intersection of a horizontal through 100 tons with the curve for 125 lb., and following a vertical through this point to the respective scales at the top of the table, we read the following values:

40.0 in. = side of a square plate required.

42.0 in. = diameter of an octagonal plate.

45.0 in. = diameter of a circular plate.

Pressures intermediate between the given curves can be easily located with sufficient accuracy.

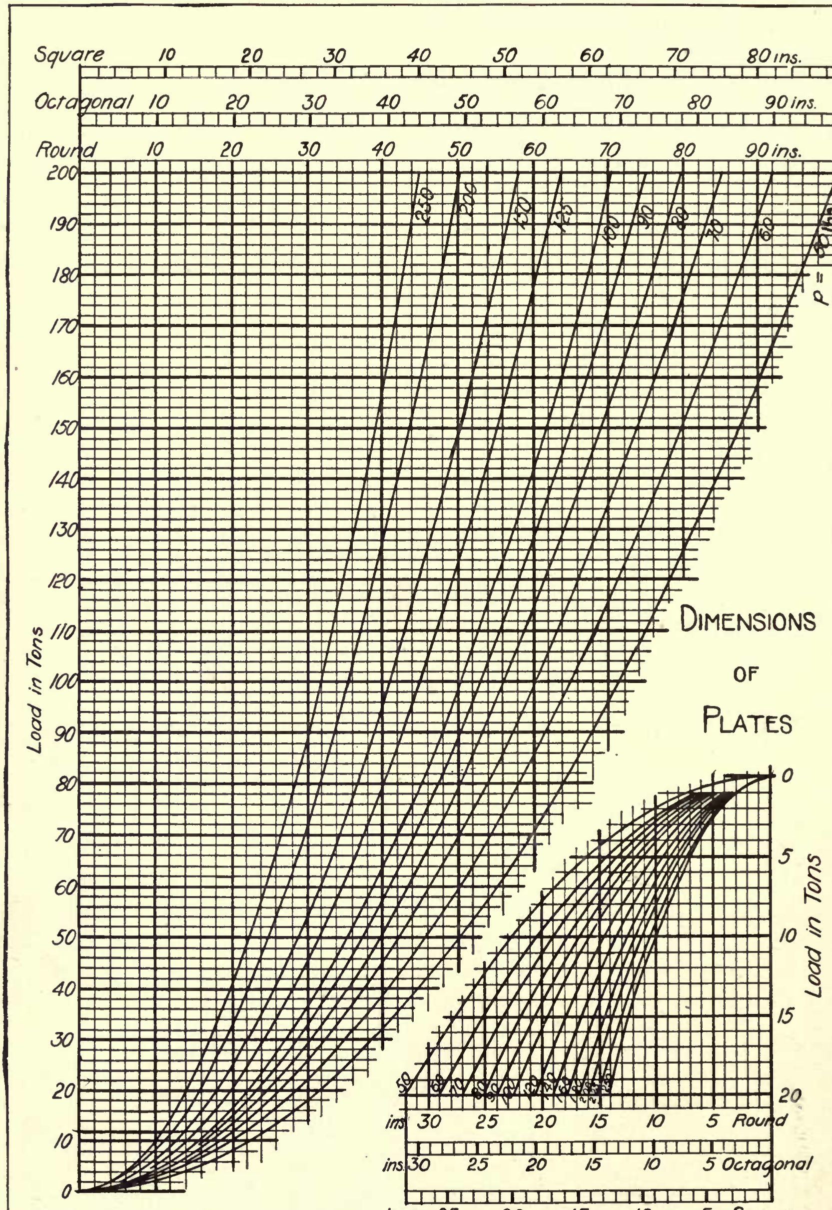

Square 10 20 30 40 50 60 70 80 ins.
Octagonal 10 20 30 40 50 60 70 80 90 ins.
Round 10 20 30 40 50 60 70 80 90 ins.
Load in Tons
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
250 200 150 125 100 90 80 70 60 p = 50 125
DIMENSIONS OF PLATES
Load in Tons
0 5 10 15 20
ins 30 25 20 15 10 5 Round
ins 30 25 20 15 10 5 Octagonal
ins 25 20 15 10 5 Square

Fig 21 Fig 22

XIV. Table for the Factor $\sqrt{\frac{3P}{B}}$

The general formula for a plate of uniform thickness is

$$t = \sqrt{\frac{3P}{fB}(L' - L'')} = \text{thickness in inches.} \tag{10}$$

This may be factored in the form

$$t = \sqrt{\frac{3P}{B}} \times \sqrt{\frac{L' - L''}{f}} = \text{thickness in inches.}$$

Fig. 23 exhibits the relations of the three quantities

$P =$ total load in tons transmitted by the plate.

$B =$ length in inches of the fracture section.

$\sqrt{\frac{3P}{B}} =$ one factor in the last formula.

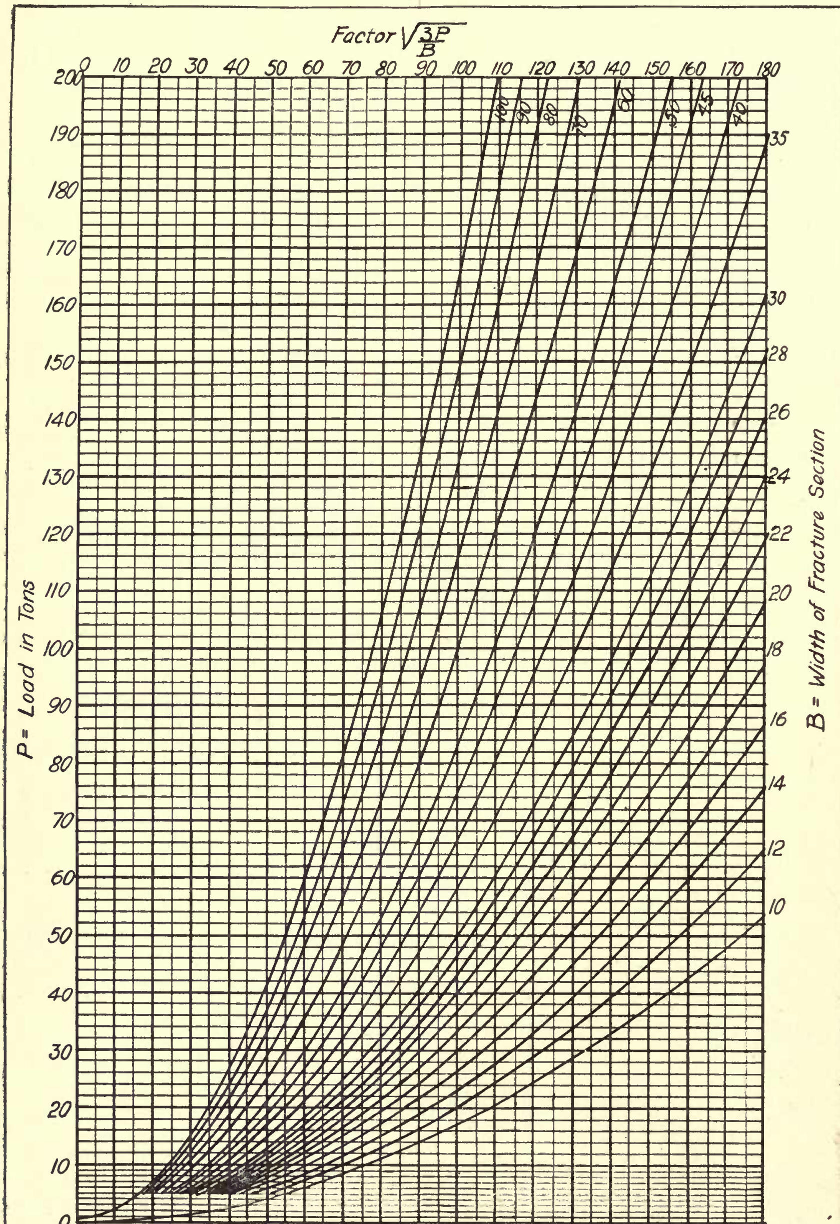
Example. Take the square plate in the last example. $P = 100$ tons, $B = 40.0$ in. The intersection of a horizontal through 100 tons with the curve for 40.0 in., and a vertical through this point gives at the top of the table $\sqrt{\frac{3P}{B}} = 122.5$.

Assume that a 12-in. cylindrical cast-iron column with 1¼-in. metal stands on this base plate.

Then $\frac{r}{R} = \frac{4.75}{6.00} = 0.79$, and by the table in Fig. 12:

$$L'' = .572 \times 6.00 = 3.43 \text{ in.}; \; L' = \frac{40}{4} = 10.0 \text{ in.}$$

Hence $L' - L'' = 10.0 - 3.43 = 6.57$ in.



Factor $\sqrt{\frac{3P}{B}}$
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180
P = Load in Tons
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
B = Width of Fracture Section
10 12 14 16 18 20 22 24 26 28 30 35 40 45 50 60 70 80 90 100

Fig. 23

XV. Tables for Plates of Uniform Thickness

The table in Fig. 24 exhibits the relations of the three quantities:

$\sqrt{\dfrac{3P}{B}}$ = the factor whose value has just been found from Fig. 23.

$L' - L''$ = difference in lever arms of the upward and downward bending moments.

t = thickness in inches represented by curved lines in the table.

In designing the tables in Fig. 24 and 25, the factor $\sqrt{\dfrac{L' - L''}{f}}$ is actually employed, but the device of separate vertical scales for the different values of f makes the table more convenient for use.

Resuming the last example and employing the table Fig. 24, a vertical through 122.5 at the top intersects a horizontal through 6.57 at the left, giving by estimation between the nearest curves the following values for t.

$t = 2\frac{1}{2}$ in. for a steel plate.
$t = 2\,^{13}/_{16}$ in. for a wrought-iron plate.

Employing the table, Fig. 25, in the same manner for cast-iron plate of uniform thickness:

$t = 5\frac{3}{4}$ in. for fiber stress of 3000 lb. per sq. in.
$t = 6\,^{5}/_{16}$ in. for fiber stress of 2500 lb. per sq. in.

Assuming 0.20 for the ratio $\dfrac{t'}{t}$, and $n = 1.17$, by the table, Fig. 16, @ = 72.5 per cent.

By formula 12,

$$t = \sqrt{\frac{100}{72.5}} \times 5.75 = 6.76 \text{ in. at middle for } f = 3000 \text{ lb. per sq. in.}$$

$$t = \sqrt{\frac{100}{72.5}} \times 6.31 = 7.41 \text{ in. at middle for } f = 2500 \text{ lb. per sq. in.}$$

Then $6.76 \times 0.2 = 1.35$ in., and $7.41 \times 0.2 = 1.48$ in., the respective thicknesses at the edges required.

It will be seen that Fig. 24 contains two vertical scales, one for steel with fiber stress of 16 000 lb. per sq. in. and the other for wrought iron with fiber stress of 12 000 lb. per sq. in. Fig. 25 likewise has two vertical scales corresponding to fiber stresses of 3000 lb. and 2500 lb. per sq. in.

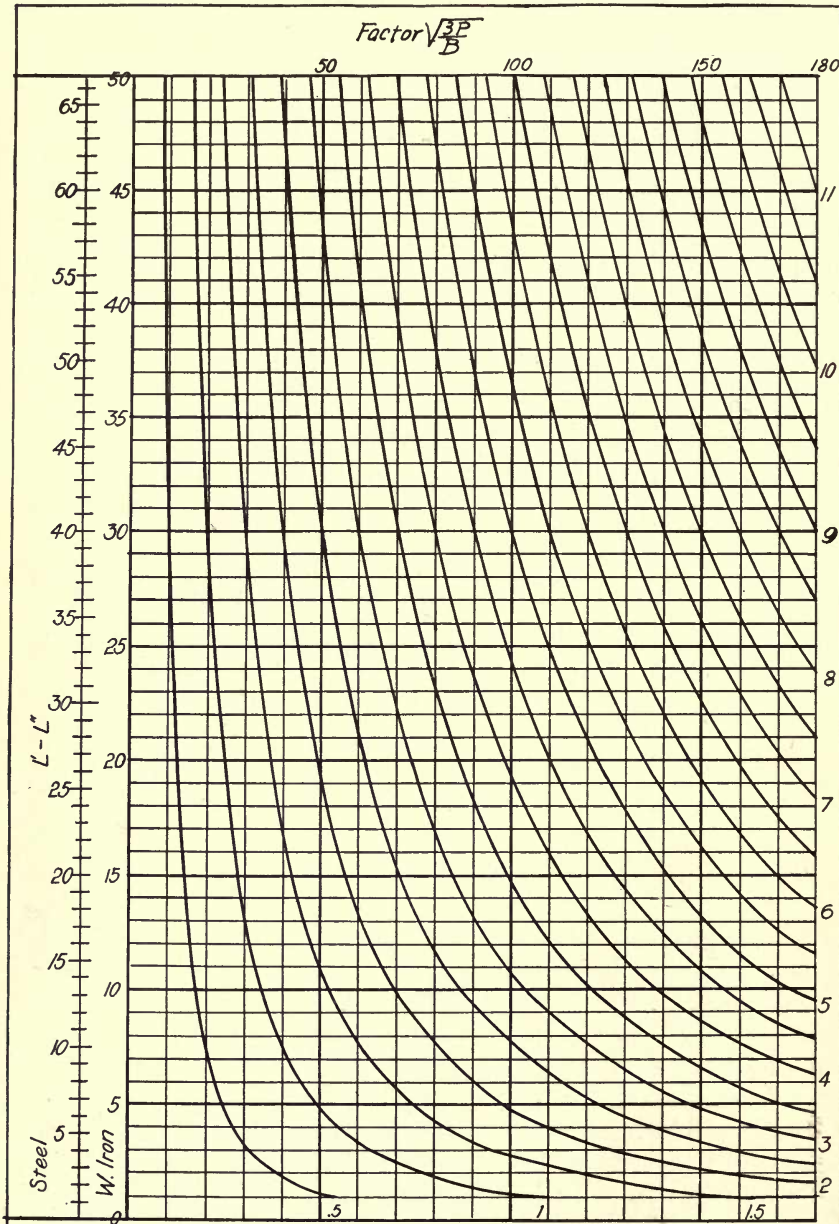

Factor $\sqrt{\frac{3P}{B}}$
50
100
150
180
L' - L''
Steel
W. Iron
.5
1
1.5
2
3
4
5
6
7
8
9
10
11

Fig. 24

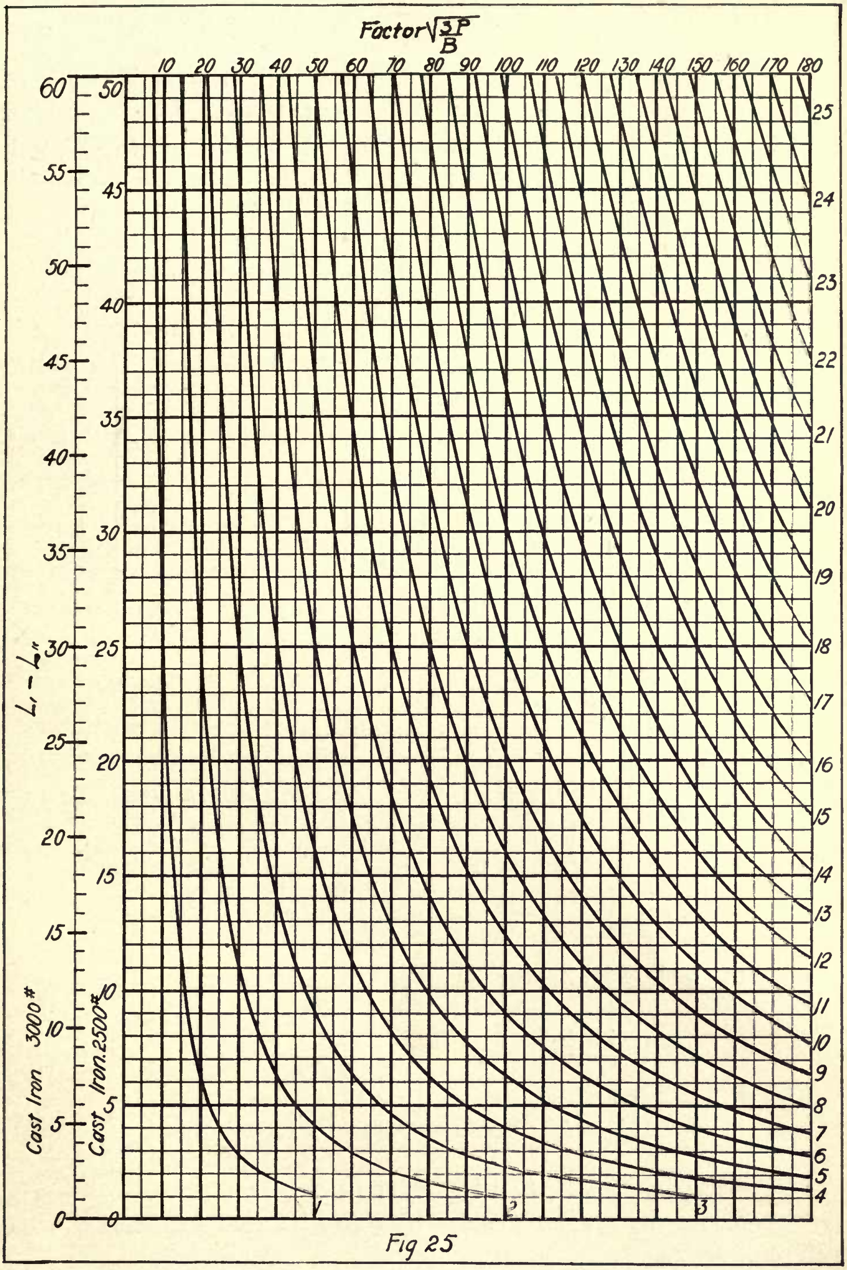

Factor $\sqrt{\frac{3P}{B}}$
10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180
$L_1 - L_{11}$
Cast Iron 3000#
Cast Iron 2500#
0 5 10 15 20 25 30 35 40 45 50 55 60
0 5 10 15 20 25 30 35 40 45 50
1 2 3
4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

Fig 25

XVI. Additional Examples of the Use of the Tables

1. A bearing plate is 12 in. wide and rests on a 13-in. wall; it supports a load of 15 tons (30 000 lb.) transmitted by the end of a girder 12 in. wide. Safe pressure of plate on masonry = 100 lb. per sq. in. Safe fiber stress for cast iron = 2500 lb. per sq. in. Required its thickness.

$$\frac{15 \times 2000}{100 \times 12} = 25.0 \text{ in.} = \text{length of the plate,}$$

$$nk = \frac{25 - 12}{2} = 6.5 \text{ in.} = \text{projection of end beyond side of girder;}$$

By formula 17,

$$t = \frac{1}{40.8}\sqrt{\frac{Pnk}{d}} = \frac{1}{40.8}\sqrt{\frac{30\,000 \times 6.5}{12.0}} =$$

3.12 in., the thickness beneath the girder. The plate may be tapered from the girder to any desired thickness at its ends without danger, since its fracture section is always rectangular.

If the safe fiber stress be taken = 3000 lb. per sq. in.;

$$t = \frac{40.8 \times 3.12}{44.7} = 2.85 \text{ in., say, } 2\tfrac{7}{8} \text{ in.}$$

Or by the table, Fig. 21, a plate 17.25 in. square would be required.

Hence, length $\dfrac{(17.25)^2}{12} = 25.0$ in., as before.

Also by the table, Fig. 23, the factor $\sqrt{\dfrac{3\,P}{B}} = 86.0.$

$$L' = \frac{25.0}{4} = 6.25\,;\; L'' = \frac{12.0}{4} = 3.00\,;$$

$$L' - L'' = 6.25 - 3.00 = 3.25 \text{ in.}$$

By the table, Fig. 25, $t = 3\tfrac{1}{8}$ in. as before.

Therefore, the graphical tables may also be used for bearing and wall plates.

2. A square base plate transmits a load of 50 tons to masonry with a safe resistance of 150 lb. per sq. in. Safe fiber stress 3000 lb. per sq. in. A column 7 in. external diameter and 1 in. thickness of metal stands on this plate.

By the table, Fig. 21, 22.7 in. = side of plate, and $n = 1.12$.

By the table, Fig. 23, $\sqrt{\frac{3P}{B}} = 114.5$.

$L' = 5.68$ in.; $L'' = 1.91$ in. by Fig. 12. $L' - L'' = 3.76$ in.

By the table, Fig. 25, $t = 3\,{}^5/_{16}$ in. $= 3.31$ in.

Assuming $\frac{t'}{t} = 0.25$, which is a commonly employed ratio, and $n = 1.12$, @ $= 73.5$ by the table in Fig. 16.

By formula 12,

$$t = \sqrt{\frac{100}{73.5}} \times 3.31 = 3.86 \text{ in.} = \text{thickness at middle, and}$$

$$t' = 3.86 \times 0.25 = 0.97 \text{ in. thickness at edges.}$$

3. A circular base plate transmits a load of 175 tons to masonry with a safe resistance of 175 lb. per sq. in. A column of metal 12 in. in diameter and 1¼ in. thick stands on this plate.

By the table, Fig. 21, 50.5 in. = diameter of the plate; $n = 1.60$.

By the table. Fig. 23, $\sqrt{\frac{3P}{B}} = 144$. Also $L' - L'' = 7.29$ in.

By the table, Fig. 24, $t = 3.10$ in. for a steel plate.

$t = 3.50$ in. for a wrought-iron plate.

By the table, Fig. 25, $t = 7.05$ in. for cast-iron plate, $f = 3000$ lb. per sq. in.

$t = 7.75$ in. for cast-iron plate, $f = 2500$ lb. per sq. in.

Assuming

$\frac{t'}{t} = 0.25$, and as $n = 1.60$, by the table, Fig. 16, @ $= 71.0$.

Then

$$t = 7.05 \times \sqrt{\frac{100}{71}} = 8.37 \text{ in.} = \text{thickness beneath column}, f = 3000 \text{ lb.}$$

$$t = 7.75 \times \sqrt{\frac{100}{71}} = 9.20 \text{ in.} = \text{same, for } f = 2500 \text{ lb.}$$

Therefore

$t' = 2.09$ in. in the first case and 2.30 in. in the second.

4. Assume that in the last case the plate is to be square and that the safe resistance of the masonry is but 90 lb. per sq. in.

In the same manner as before, we easily find:

Side of plate = 62.2 in.; $n = 2.09$.

Factor $\sqrt{\frac{3P}{B}} = 130$

$t = 3\frac{5}{8}$ in. for a steel plate
$t = 4\frac{3}{4}$ in. for a wrought-iron plate
$t = 8\frac{3}{8}$ in. for cast-iron plate, $f = 3000$ lb.
$t = 9\frac{1}{8}$ in. for same with $f = 2500$ lb.

Assuming $\frac{t'}{t} = 0.25$, and for $n = 2.09$; @ = 69.5.

Then $t = 10.00$ in. thickness at middle in the first case and = 10.95 in the second. The corresponding values for t' are 2.50 and 2.74 inches.

5. A built steel column is composed of two 15-in. channels, one 15-in. I, and two 16 x ¾-in. plates. It stands on a circular cast-iron base plate, which rests directly on a cylindrical sunken foundation pier of portland cement concrete, to which the plate transmits a load of 500 tons (1 000 000 lb.) Maximum safe fiber stress in cast iron is taken at 2500 lb. per sq. in. Safe resistance of the concrete pier is 175 lb. per sq. in. Let $\frac{t'}{t} = 0.20$.

Required least safe diameter and thickness of the cast-iron plate.

In this case, $\frac{1\,000\,000}{175} = 5714.3$ sq. in., area of plate.

$$d = \sqrt{\frac{5714}{.7854}} = 85.3 \text{ in.} = \text{diameter of the plate}; n = 2.17.$$

By formula Sec. VII, $L' = 85.3 \times 0.2122 = 18.1$ in.

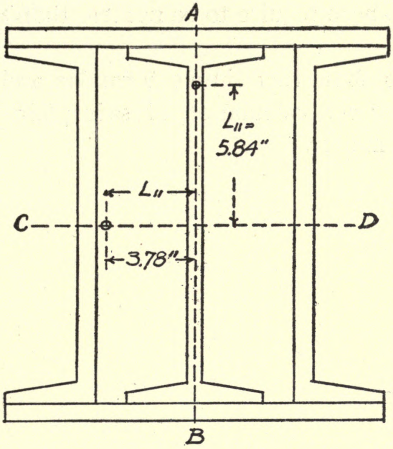


FIG. 26

For the given cross section of the steel column, Fig. 26, there may readily be found by the usual graphical methods:

$$L'' = 5.84 \text{ in. about the axis } CD.$$

$$L'' = 3.78 \text{ in. about the axis } AB.$$

Taking the smaller of these values and applying formula 32,

$$t = \frac{1}{28.9}\sqrt{\frac{P}{B}(L' - L'')} = \frac{1}{28.9} \times$$

$$\sqrt{\frac{1\,000\,000}{85.3}(18.10 - 3.78)} = 14.18 \text{ in.}$$

Assuming $\frac{t'}{t} = 0.20$, and for $n = 2.17$, by table in Fig. 16,

$$@ = 68.0$$

Then $t = \sqrt{\frac{100}{68.0}} \times 14.18 = 17.20$ in. = thickness under column, and $t' = 17.2 \times 0.20 = 3.44$ in. = thickness at edge of plate.

Such a solid plate would be more simple and more easily set in place, and it might also be cheaper than the usual arrangement consisting of a cast-iron ribbed base plate or stool above a layer of short steel 15-inch I-beams, which are set on the top of the concrete pier. This pier would also here require to be not less than 8 ft. 6 in. in diameter at the top.

These examples show that simple formulas and tables have been here devised, making the calculation of safe plain bearing and base plates a very simple matter.

Publications of The Engineering Experiment Station

Bulletin No. 1. Tests of Reinforced Concrete Beams, by Arthur N. Talbot. 1904. (*Out of print*).

Circular No. 1. High-Speed Tool Steels, by L. P. Breckenridge. 1905. (*Out of print*).

Bulletin No. 2. Tests of High-Speed Tool Steels on Cast Iron, by L. P. Breckenridge and Henry B. Dirks. 1905. (*Out of print*).

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Bulletin No. 4. Tests of Reinforced Concrete Beams, Series of 1905, by Arthur N. Talbot. 1906.

Bulletin No. 5. Resistance of Tubes to Collapse, by Albert P. Carman. 1906. (*Out of print*).

Bulletin No. 6. Holding Power of Railroad Spikes, by Roy I. Webber. 1906. (*Out of print*).

Bulletin No. 7. Fuel Tests with Illinois Coals, by L. P. Breckenridge, S. W. Parr and Henry B. Dirks. 1906. (*Out of print*).

Bulletin No. 8. Tests of Concrete: I. Shear; II. Bond, by Arthur N. Talbot. 1906. (*Out of print*).

Bulletin No. 9. An Extension of the Dewey Decimal System of Classification Applied to the Engineering Industries, by L. P. Breckenridge and G. A. Goodenough. 1906.

Bulletin No. 10. Tests of Concrete and Reinforced Concrete Columns, Series of 1906, by Arthur N. Talbot. 1907. (*Out of print*).

Bulletin No. 11. The Effect of Scale on the Transmission of Heat through Locomotive Boiler Tubes, by Edward C. Schmidt and John M. Snodgrass. 1907. (*Out of print*).

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Bulletin No. 15. How to Burn Illinois Coal without Smoke, by L. P. Breckenridge. 1908.

Bulletin No. 16. A Study of Roof Trusses, by N. Clifford Ricker, 1908.

Bulletin No. 17. The Weathering of Coal, by S. W. Parr, N. D. Hamilton and W. F. Wheeler. 1908. (*Out of print*).

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